Chapter 2
Failure Modes and Wedge Sliding Analysis

2-1. General

The objective of a stability analysis is to maintain horizontal, vertical, and rotational equilibrium of the structure. Geotechnical information is needed to properly define and perform a realistic stability analysis. Possible failure modes and planes of weakness must be determined from onsite conditions, material strengths, and uplift forces. Stability is ensured by:

- Providing an adequate factor of safety against sliding at all possible failure planes.
- Providing specific limitations on the magnitude of the foundation bearing pressure.
- Providing constraints on the permissible location of the resultant force on any plane.
- Providing an adequate factor of safety against flotation of the structure.

However, satisfying the above provisions may not ensure stability if the structure experiences significant loss of foundation material due to erosion or piping, or if there is an internal failure due to inadequate strength of the structural materials. Stability is just one of the requirements necessary to ensure adequate structural performance.

2-2. Limit Equilibrium Analysis

The forces and pressures acting on a structure are indeterminate. Static equilibrium equations are insufficient to obtain a solution for lateral soil forces; additional assumptions must be incorporated in the analysis. For nonlinear materials, such as soils, this is commonly done by assuming that a limit or failure state exists along some surface and that the shear force along the surface corresponds to the shear strength of the material. With these assumptions, equilibrium equations can be solved. Hence, this approach is commonly called limit-equilibrium analysis. To ensure that the assumed failure does not occur, a reduction factor (safety factor or strength mobilization factor) is applied to the material strength. It should be noted that this approach differs significantly from that commonly used for indeterminate structure analysis, where stress-strain properties and deformations are employed. This limit equilibrium approach provides no direct information regarding deformations; it is implied that deformations are sufficient to induce the failure condition. Actual deformations will vary non-linearly in response to actual applied loads. Deformations are indirectly limited to tolerable values by the judicious selection of a safety factor.

2-3. Sliding Planes

Stability must be assessed on selected surfaces within structure in accordance with the methods presented in EM 1110-2-2200. Sliding safety must also be assessed at/or near the foundation-structure interface. This surface may be either level or sloping. Generally, it may be assumed that a surface that slopes upward (in the direction of possible sliding) will have a beneficial effect, while one that slopes downward will increase the possibility for sliding. Figure 2-1 illustrates the beneficial and adverse effects of base slope. Where a shallow weak seam exists below a structure's contact with the foundation, or a structure is imbedded below the top of the foundation, two possible failure modes are present. One mode involves slippage along the weak plane (directly under the structure) and along its extension until it daylights. The other mode involves slippage along the weak plane directly under the structure plus slippage along a plane through the foundation above the weak seam (crossbed shear for rock or passive resistance for soil). When the weak seam extends a large distance past the toe of the structure without daylighting, the second mode will usually be critical. Figure 2-2 illustrates these modes of failure.

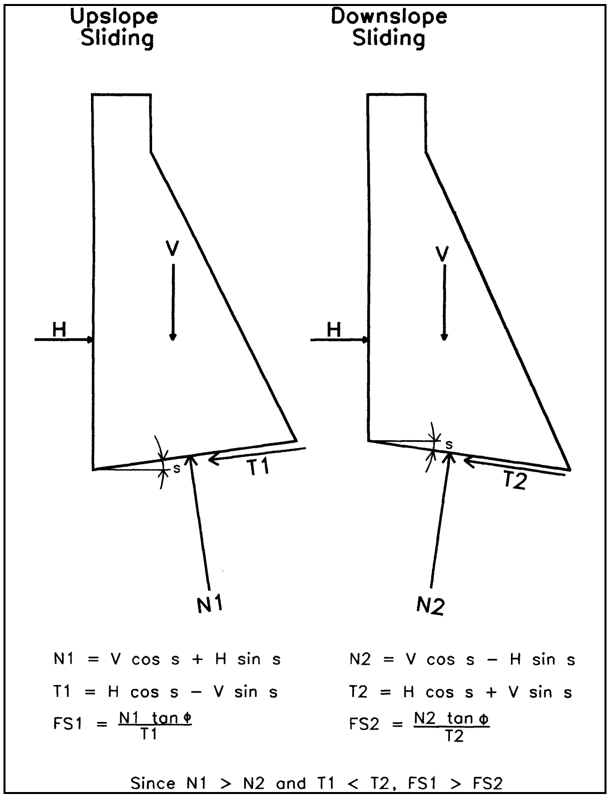

Figure 2-1

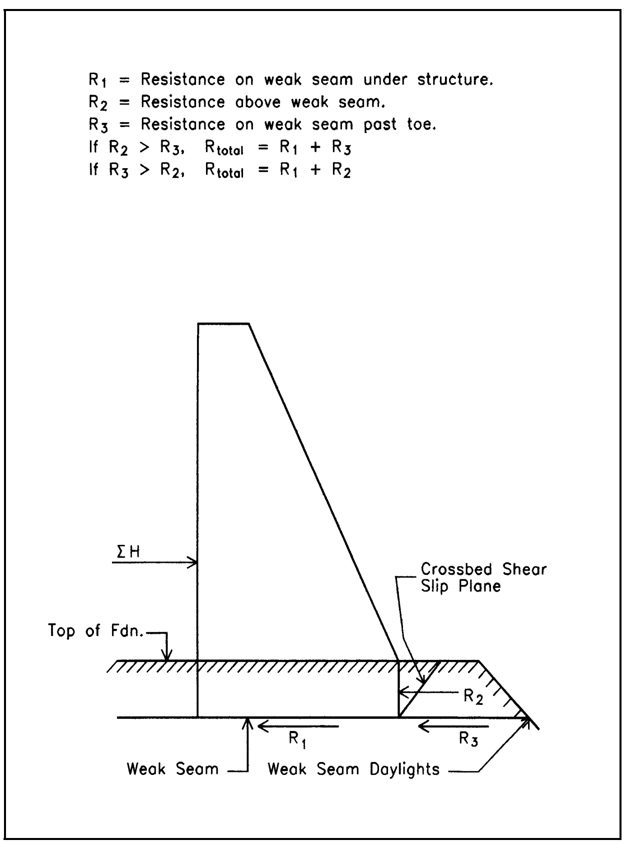

Figure 2-2

2-4. Resultant Location

Conformance with resultant location requirements ensures that the structure is safe from rotational failure. The slope of the resultant and its location are critical in assessing the foundation bearing capacity. For some load condition categories, the resultant is allowed to fall outside the middle-third of the base. In these instances, it is assumed that the structure-foundation interface has no capability for resisting tensile stresses; therefore, part of the structure's base is assumed to lose contact with the foundation resulting in changes to the uplift pressure acting on the base.

2-5. Flotation

This mode of failure occurs when the net uplift force (gross uplift on the base minus the weight of surcharge water above the structure) exceeds the summation of forces due to the weight of the structure, the weight of water contained in the structure, and other surcharge loads.

2.6. Bearing

Analytical methods, traditional bearing capacity equations, field tests, and laboratory tests are all used to determine the bearing capacity of soil and rock. The allowable bearing capacity is defined as the maximum pressure that can be permitted on a foundation soil or rock mass giving consideration to all pertinent factors, with adequate safety against rupture of the soil or rock mass, or settlement of the foundation of such magnitude as to jeopardize the performance and safety of the structure. Increases in allowable bearing capacity are permitted for unusual and extreme load conditions over those required for usual load conditions. The allowable increases are provided in Chapter 3. Shear strength parameters used in the determination of bearing capacity values shall be established in accordance with the discussion presented in Paragraph 2-8.

a. *Soil.* For structures founded on soil, the bearing capacity is limited by the ability of the soil to safely carry the pressure placed on the soil from the structure without undergoing a shear failure. Prevention of a shear failure, however, does not ensure that settlements will be within acceptable limits, therefore, a settlement analysis is usually performed in addition to the shear analysis. Discussion on methods for estimating settlements and limitations in accuracy of settlement analyses is contained in EM 1110-1-1904. Methods for determining allowable bearing capacity of soils are covered in EM 1110-1-1905. General shear failure in a homogeneous soil foundation, for a vertical loading applied at the middle of a structure with a horizontal base-foundation contact, is illustrated by Prandtl's arc of shear failure as shown in Figure 2-3. The shape of the failure surface will be affected by eccentricity, the presence of shear components, and slope of the contact surface. Eccentricity and the presence of shear components will tend to make this type of failure more probable, while a sloping contact surface can either increase or decrease the probability of failure depending upon the slope direction.

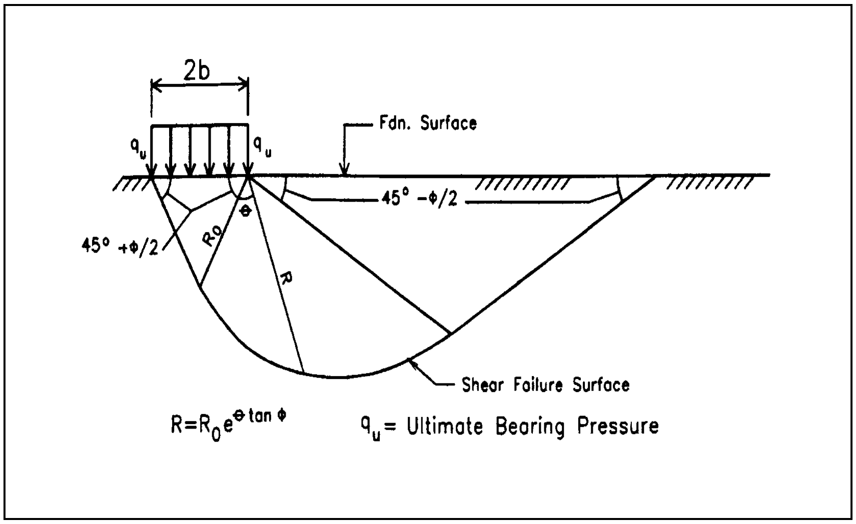

Figure 2-3

b. *Rock.* For structures founded on rock, failure modes may consist of local crushing, shear failures on weak seams, and failures at discontinuities or along bedding planes. The bearing capacity of rock will depend on whether the rock is intact, jointed, layered, or fractured. Methods used for determination of rock bearing capacities are contained in EM 1110-1-2908. The design for structures on rock foundations will involve sliding stability analyses as well as bearing capacity and settlement analyses. Sliding stability analyses address the ability of the rock foundation to resist the imposed loads without the occurrence of shearing on any horizontal or sloping weak plane. Basic rock foundation data that should be obtained for use during the design stage include material properties, strike, dip, thickness, and discontinuities such as faults, fissures and fractures. Such information should be incorporated into the bearing capacity, settlement, and sliding stability analyses.

c. *Coordination between disciplines.* The structural engineer and geotechnical engineer/geologist must coordinate their efforts in order to properly evaluate the bearing capacity of a foundation. Bearing capacity is affected by the size and shape of structure's base, the type of structure, type of loading (static or dynamic), load duration, the eccentricity of the load acting on the foundation, and the shear components of the load; all of which should be furnished to the geotechnical engineer/geologist by the structural engineer. The location and identification of weak zones and planes or discontinuities, soil and rock strength parameters, information on existing faults, and the allowable bearing capacity of the foundation should be furnished to the structural engineer by the geotechnical engineer/geologist.

2-7. Geotechnical Explorations and Testing

The scope of any geotechnical investigation will depend on geological structural complexity, imposed or existing loads acting on the foundation, and to some extent the consequences should a failure occur. The complexity of the foundation will determine the extent of drill holes, mapping, trenching, and other exploratory measures, which may be required to accurately describe foundation conditions. Guidelines for foundation explorations and testing are provided in EM 1110-1-1802, EM 1110-1-1804, and EM 1110-1-2908.

2-8. Shear Strength Tests

Shear strength parameters required for bearing capacity and sliding stability analyses may be estimated for soils from the results of in situ tests and/or by direct shear and triaxial tests performed in the laboratory. For rock, these values are usually obtained from laboratory tests. Shear strength is a function of many complex independent variables including mineralogy; particle size, shape and gradation; cementation; degree of consolidation; state of stress; anisotropy; and drainage conditions. Therefore, any tests performed in the laboratory should model the conditions that will occur during project operation. Since shearing may take place on any plane that includes intact rock, sheared rock, or jointed rock, strength values for all differing rock conditions must be established in order that sliding stability and bearing capacity may be determined. EM 1110-2-1906 provides guidance on laboratory soils testing. Procedures for testing soils are also described in EM 1110-1-1804 and EM 1110-2-1913. Procedures for testing rock specimens are given in EM 1110-1-2908 and the "Rock Testing Handbook" (U. S. Army Waterways Experiment Station 1980). Coordination must be maintained between the structural engineer and the geotechnical engineer/geologist to ensure that safe and economical designs are obtained.

a. Soils Tests.

(1) *In-situ soils tests.*

- Standard Penetration Test (SPT). The SPT resistance, often referred to as the blow count, is frequently used to estimate the relative density of soil. This relative density can then be correlated with the angle of internal friction, ϕ, and the undrained shear strength, c.

- Cone Penetration Test (CPT). The CPT may also be used to estimate the relative density of cohesionless soils and the undrained shear strength of cohesive soils. The CPT is especially suitable for sands, where it is preferable to the SPT.

- Field Vane Test (FVT). The FVT is commonly used to estimate the in situ undrained cohesive strength of soft to firm clays.

(2) Laboratory soils tests.

- Q-Test. In a Q test the water content of the soil sample is not permitted to change either prior to or during load application. The Q test produces results that approximate the shear strength available for short-term loading conditions. In cohesive soils this test yields relatively large c (cohesion) values and very low or zero ϕ values.

- R-Test. The R test represents conditions in which impervious or semi-pervious soils that have been consolidated under one set of stresses are subjected to a stress change without time for further change in water content prior to failure. In cohesive soil this test furnishes undrained shear strength parameters.

- S-Test. An S test is used to measure drained or effective stress strength parameters, c' and ϕ'. The soil sample is consolidated under an initial confining stress and loading increments are applied slowly enough to allow pore water pressures to dissipate with each load increment during shear. Results of S tests are applicable to free-draining soils where excess pore pressures do not develop during shear. S tests are also used for evaluating the shear strength of cohesive soils under long term loading conditions, where excess pore pressures have dissipated.

b. Rock Tests. The tests used for evaluating rock shear strength parameters should mirror, as closely as possible, the conditions that are expected to exist in the field. The structural engineer is referred to EM 1110-1-2908, and numerous references contained therein, in order to become familiar with such things as rock descriptors, rock mass classification systems, laboratory classification and index tests for rock, selection of modulus of deformation, etc.

(1) In-situ rock tests.

- In-Situ Direct Shear Test. In-situ direct shear tests are expensive and are only performed where critically located, thin, weak, continuous seams exist within relatively strong adjacent rock. Relatively large surface areas must be tested in order to address unknown scale effects. The test, as performed on thin, fine grained, clay seams, is considered to be an undrained test.

- In-Situ Uniaxial Compression Test. In-situ uniaxial compressive tests are expensive. This test is used to measure the elastic properties and compressive strength of large volumes of virtually intact rock in an unconfined state. The results obtained are useful in evaluating the effects of scale, however, the test is seldom used just for this purpose.

(2) Laboratory rock tests.

- Unconfined Uniaxial Compression Test. This test is performed primarily to obtain the unconfined compression strength and the elastic properties of a rock sample. Poisson's ratio can be determined if longitudinal and lateral strain measurements are taken during the test. Occasionally samples are tested with differing orientations in order to describe three-dimensional anisotropy. This test may not be indicative of the overall rock mass strength.

- Triaxial Test. The triaxial test can be made on intact, cylindrical rock samples. Data is provided for determination of rock strength in the drained or undrained state when subjected to three-dimensional loading. Data from the test, used in calculations, provides the strength and elastic properties of the sample at various confining pressures. Strengths along planes of weakness, such as natural joints, seams, and bedding, can be determined if these planes are properly oriented in the test. The oriented plane variation is particularly useful for strengths on thinly filled discontinuities containing soft materials. Since clean discontinuities are free draining, tests performed on them should be drained tests. Tests on discontinuities filled with coarse-grained materials should also be drained tests. The tests for discontinuities filled with fine-grained materials should be undrained tests.

- Laboratory Direct Shear Test. The laboratory direct shear test is primarily used to measure the shear strength, at various normal stresses, along planes of discontinuity or weakness. When this test is performed on the surface of a clean discontinuity (with asperities) that is subjected to very high normal stress, with a rapidly applied shear stress and small deformations, the values obtained will represent the *peak shear strength*. The test is often performed at a reduced rate of shear stress application, with intermediate or low normal stresses, and with the asperities over-ridden, resulting in reduced values for shear strength. Repetitive shearing of a sample, or continuing displacement to a point where shear strength is constant, establishes the *residual shear strength* that is available. For the test results to be valid, test conditions must be as close as possible to the conditions that will exist in the field. Test drainage conditions should be essentially the same as for the triaxial tests discussed above. *Upper bound* and *lower bound* shear strengths are discussed in EM 1110-1-2908.

2-9. Selection of Design Shear Strengths

Design shear strength parameters should be selected by the geotechnical engineer/geologist in consultation with the structural engineer. All parties must be aware of the implicit assumptions pertaining to the stability analysis procedure that is being used. The design strengths should be selected such that they do not result in uneconomical, ultra-conservative designs. However, in certain instances it may be more economical to assume conservative design shear strength parameters than to institute an expensive testing program. Selected strength parameters must be appropriate for the actual stress states and drainage conditions expected for the foundation materials. Laboratory results are dependent on the details of the testing procedures and the condition of the samples tested. The conversion from laboratory test data to in-situ strength parameters requires careful evaluation. A combination of experience and judgment is required to give the level of confidence needed for selecting the strength parameters. The shear strength of intact rock as well as rock with clean or filled discontinuities is dependent on many factors including confining pressures, loading history, and rate of loading. Specimen size is also a factor which must be considered when estimating shear strength based on laboratory testing. The number, orientation, and size of discontinuities and weaknesses may vary considerably, thus affecting load distribution and the final results. When selecting design shear strengths, the shape of stress-strain curves for individual tests should be considered. Where undisturbed and compacted samples do not show a significant drop in shear or deviator stress after the peak stress is reached, the design strength can be chosen as the peak shear stress. Where significant differences in stress-strain characteristics exist along a potential failure surface, stress-strain compatibility and the potential for progressive failure must be considered in selection of design strength parameters. With varied foundation conditions, it may not be possible to have all the foundation materials at their peak strengths at the same displacement (see Chapter 6). In those conditions, and for conditions that rely on passive resistance of a rock wedge or soil backfill, the strength values must be consistent with the displacements that will put the structure at the limit state assumed for the sliding stability analysis. A discussion of the sliding equilibrium method and its limitations can be found in EM 1110-1-2908.

2-10. Multiple-Wedge Sliding Analysis

a. Basic concepts. The multiple-wedge sliding analysis is a fairly simple assessment of the sliding factor of safety along various potential sliding planes, that can also account for the behavior expected from complex soil stratification and geometry. It is based on modern principles of structural and geotechnical mechanics that apply a safety factor to the material strength parameters in a manner, which places the forces acting on the structure and foundation wedges in sliding equilibrium. This method of analysis is illustrated in Appendix D, example D2. Derivation of the governing equilibrium equation for a typical wedge is shown in Appendix E. See EM 1110-1-2908 for additional information on this method. Following are the principles, assumptions and simplifications used in multiple-wedge sliding analysis.

- Sliding stability of most concrete structures can be adequately assessed by using a limit equilibrium approach.

- A sliding mode of failure will occur along a presumed failure surface when the applied shearing force exceeds the resisting shearing forces.

- The failure surface can be any combination of plane and curved surfaces, but for simplicity, all failure surfaces are assumed to be planes, which form the bases of wedges.

- Analyses are based on assumed-plane failure surfaces. The calculated safety factor will be realistic only if the assumed failure mechanism is possible.

- The factor of safety is defined, and minimum required factors of safety are given in Chapter 3.

- The lowest safety factor on a given failure surface can be determined by an iterative process. However, a single-step analysis using the required minimum factor of safety, can be used as a simple pass/fail test.

- A two-dimensional analysis is presented in this manual. These principles should be extended if unique three-dimensional geometric features and loads critically affect the sliding stability of a specific structure.

- Only force equilibrium is satisfied in this analysis, moment equilibrium is not ensured.

- The shearing force acting along the vertical interface between any two wedges is assumed to be negligible. Therefore, the failure surface at the bottom of each wedge is only loaded by the forces directly above it.

- A linear relationship is assumed between the resisting shearing force and the normal force acting on the failure surface beneath each wedge.

- The maximum shear strength that can be mobilized is adequately defined by the Mohr-Coulomb failure theory.

- Considerations regarding displacements are excluded from the limit equilibrium approach. The relative rigidity of different foundation materials and the concrete substructure may influence the results of the sliding-stability analysis. Such complex structure-foundation systems may require a more intensive sliding investigation than a limit equilibrium approach. The effects of strain compatibility along the assumed failure surface may be included by interpreting data from in situ tests, laboratory tests, and finite element analyses.

b. Analytical procedure. Following is a general procedure for analyzing multi-wedge systems.

- Assume a potential failure surface which is based on the stratification, location and orientation, frequency and distribution of discontinuities of the foundation material, and the configuration of the substructure. Discontinuities in the slip path beneath the structural wedge should be modeled by assuming an average slip plane along the base of the structural wedge. The structural wedge may include rock or soil that lies below the base of the concrete structure.

- Divide the assumed slide mass into a number of wedges. Since all portions of the structure must slide as a unit, there can be only a single structural wedge.

- The interface between the group of driving wedges (wedges with negative slip plane inclination angles) and the structural wedge is assumed to be a vertical plane located at the heel of the structural wedge and extending to the base of the structural wedge. The magnitudes of the driving forces depend on the actual values of the safety factor and the inclination angles (α) of the slip path. The inclination angles, corresponding to the maximum driving forces for each potential failure surface, can be determined by independently analyzing the group of driving wedges for a trial safety factor. In rock, the inclination may be predetermined by discontinuities in the foundation. The general equation applies to wedges with lateral earth forces that act horizontally.

- The interface between the group of resisting wedges (wedges with positive slip plane inclination angles) and the structural wedge is assumed to be a vertical plane located at the toe of the structural wedge and extending to the base of the structural wedge. The magnitudes of the resisting forces depend on the actual values of the safety factor and the inclination angles of the slip path. The inclination angles, corresponding to the minimum resisting forces for each potential failure mechanism, can be determined by independently analyzing the group of resisting wedges for a trial safety factor. When resisting force is used, special considerations may be required. Rock that may be subjected to high velocity water scouring should not be used unless adequately protected. Also, the compressive strength of the rock layers must be sufficient to develop the wedge resistance. In some cases, wedge resistance should not be assumed without resorting to special treatment such as installing rock anchors.

- Draw free body diagrams which show all the forces assumed to be acting on each wedge. The orientation of the failure surfaces for most wedges can be calculated directly by using the equations in paragraph 5-4.

- The analysis proceeds by assuming trial values of the safety factor and unknown inclinations of the slip path so the governing equilibrium conditions, failure criterion, and definition of safety factor are satisfied. An analytical or a graphical procedure may be used for this iterative solution.

- If it is only necessary to determine whether an adequate safety factor exists, this may be determined in a single step without the iterative process.

- For some load cases, the normal component of the resultant applied loads will lie outside the kern of the base area, and a portion of the structural wedge will not be in contact with the foundation material. The sliding analysis should be modified for these load cases to reflect the following secondary effects due to coupling of sliding and rotational behavior. The uplift pressure on the portion of the base, which is not in contact with the foundation material, should be a uniform value, which is equal to the hydrostatic pressure at the adjacent face, (except for instantaneous load cases such as due to seismic forces). The cohesive component of the sliding resistance should only include the portion of the base area, which is in contact with the foundation material.

c. Coordination. An adequate assessment of sliding stability must account for the basic structural behavior, the mechanism of transmitting compressive and shearing loads to the foundation, the reaction of the foundation to such loads, and the secondary effects of the foundation behavior on the structure. A fully coordinated team of geotechnical and structural engineers and geologists should ensure that the result of the sliding analyses is properly integrated into the overall design. Some of the critical aspects of the design process which require coordination are:

- Preliminary estimates of geotechnical data, subsurface conditions, and types of structures.
- Selection of loading conditions, loading effects, potential failure mechanisms, and other related features of the analytical models.
- Evaluation of the technical and economic feasibility of alternative structures.
- Refinement of the preliminary substructure configuration and proportions to consistently reflect the results of detailed geotechnical site explorations, laboratory testing, and numerical analyses.
- Modification of the structure configuration or features during construction due to unexpected variations in the foundation conditions.

2-11. Single-Wedge Sliding Analysis

Only the structural wedge is actively considered in the single-wedge sliding analysis. This is a simpler method, which will produce satisfactory results. The basic concepts are similar for both the single and multiple-wedge methods, but all driving and resisting wedges are replaced with earth and groundwater forces calculated directly, using the methods in Chapter 5. The single-wedge method is illustrated in Appendix D, example D1. This method produces reasonably conservative estimates of the earth forces used for the sliding analysis and for other stability analyses and for structural design.

2-12. Mandatory Requirements

There are no mandatory requirements in this chapter.